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16. ABSTRACT

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The specific objectives of the research program are developed on a yearly basis through discussions between the staffs of the Pavements Section of the Materials and Research Department and of the Laboratory at the University. During the first year it was intended that the investigation attempt to define the effects of asphalt type and hardness and aggregate type on the behavior of mixtures in repeated flexure.

The asphalts to be investigated included:

1. One, 85-100 penetration asphalt cement (low penetration ratio).
2. One, 85-100 penetration asphalt cement (used in asphalt concrete pavements at AASHO Road Test).
3. Three asphalt cements from one source (medium penetration ratio):
   a. 120-150 pen.
   b. 85-100 pen.
   c. 40-50 pen.

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SOIL MECHANICS AND BITUMINOUS MATERIALS RESEARCH LABORATORY

ASPHALT MIXTURE BEHAVIOR IN REPEATED FLEXURE

by

C. L. MONISMITH

INSTITUTE OF ENGINEERING RESEARCH
REPORT NO. TE-63-2 [redacted]
to
THE MATERIALS AND RESEARCH DEPARTMENT
DIVISION OF HIGHWAYS
STATE OF CALIFORNIA

DEPARTMENT OF CIVIL ENGINEERING
INSTITUTE OF TRANSPORTATION AND TRAFFIC ENGINEERING

TE63-2

[redacted]

Soil Mechanics and Bituminous Materials
Research Laboratory

ASPHALT MIXTURE BEHAVIOR IN REPEATED FLEXURE

A Report on an Investigation
by C. L. Monismith
Associate Professor of Civil Engineering

to

The Materials and Research Department
Division of Highways
State of California
Under
State of California Standard Agreement MR-127

Report No. TE-63-2
Institute of Engineering Research
University of California
Berkeley, California
November 1963

INTRODUCTION

To better define the mix variables which influence the fatigue characteristics of asphalt concrete and possibly to develop, eventually, a failure criterion which will adequately describe the fatigue mechanism in asphalt mixtures, a four-year program is now underway at the Soil Mechanics and Bituminous Materials Laboratory of the University of California. This program is financed by a research grant from the Materials and Research Department of the California Division of Highways.

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The asphalts to be investigated included:

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3. Three asphalt cements from one source (medium penetration ratio):
   a. 120-150 pen.
   b. 85-100 pen.
   c. 40-50 pen.

These materials, with the exception of the AASHO asphalt, were supplied by the Materials and Research Department and were obtained from California productions.

The aggregate types to be investigated included:

1. Crushed granite from Watsonville, California.
2. Crushed gravel from Sacramento, California.

The purpose of this report is to summarize the results of this first year's effort. It will be noted that about one-half of the intended study utilizing the project materials was completed. The availability of data from another project, however, has made it possible to include indications of the effects of all of the desired variables.

While the primary objective of the research is to study the fatigue characteristics of asphalt concrete, the range in materials being investigated made desirable a secondary objective, namely, the development of thermal stress due to restraint of deformation in asphalt concrete which would arise from daily temperature changes. Some data pertaining to this objective are also presented in this report.

MATERIALS AND SPECIMENS

As noted in the introduction, while it was originally planned that two different aggregate types would be tested, time limitations permitted the utilization of one aggregate, the crushed granite from Watsonville, California.

This material has a uniform apparent specific gravity of 2.92. Previous tests on the material using the Rice vacuum saturation technique* indicated an effective specific gravity of 2.92 also. The grading, as defined in the original proposal, was to conform to the 1/2-in. maximum, medium gradation of the 1960 State of California Standard Specifications. A plot of the actual grad-

*Rice, J. M., "Volumetric Methods for Measuring Asphalt Content and Effective Gravity of Aggregates in Bituminous Mixtures", Proceedings, AAPT, Vol. 22, 1953, p. 284.

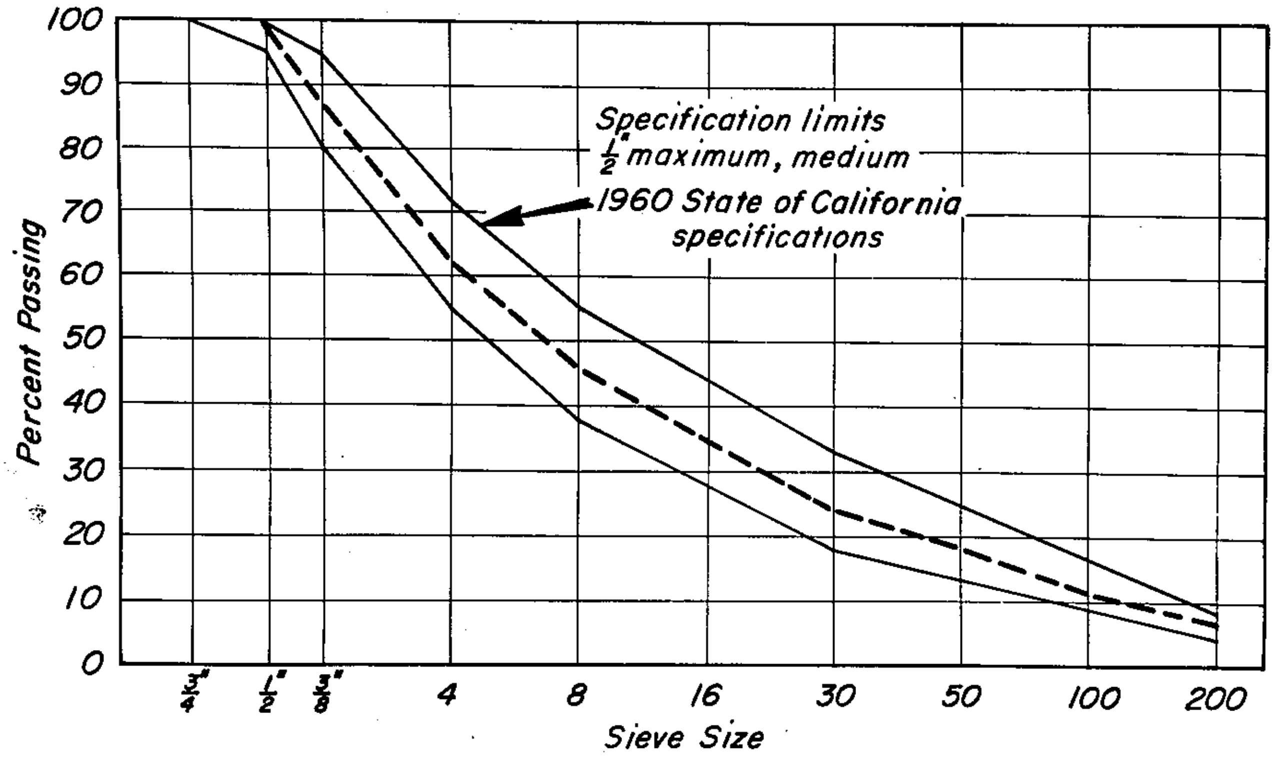

Fig. 1 — Grading curve Watsonville aggregate.

ing, together with specification limits, is shown in Fig. 1. To insure uniformity in the gradation of the aggregate, all the material was separated into individual sizes by screening and then recombined to obtain the desired proportions.

Five asphalts were to be investigated. Four of the materials were supplied by the Materials and Research Laboratory and are representative of California asphalts. The fifth material is a sample of the asphalt used in the AASHO Road Test and supplied directly to the University by the test road staff. This material was sampled from the feeder line of one of the plants at the time of the paving operation on the project. Table 1 contains results of 1960 Standard Specification tests performed on the California asphalts by the Materials and Research Laboratory staff. Since the sample of the AASHO material supplied from our stock was insufficient to permit the 1960 standard tests, other than penetration, to be performed, included for the AASHO asphalt are average data obtained by the test road staff on twelve samples of the material obtained from the two refinery tanks. Table 2 lists test results for the five asphalts based on tests required in the new tentative specifications published in 1962.*

For convenience the asphalts will, in the remainder of the report, be referred to by letter. The coding adopted is shown in Table 3. Also shown in this table is a comparison of the results of penetration tests performed on the original samples by the staff of the University Laboratory and by the State Laboratory.

In this investigation, asphalt content was not intended to be a variable. Thus the mixtures containing each of the asphalts were tested in fatigue at an asphalt content which would be representative of field conditions. This asphalt content was selected utilizing the mix design procedure of the State of California** and the 1960 standard specifications for a Type B aggregate.

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* Hveem, F. N., E. Zube and J. Skog, "Proposed New Tests and Specifications for Paving Grade Asphalts", Proceedings, AAPT, Vol. 32, 1963, pp. 271-328.

** California Test Method Nos. 304C, 305B, and 306B.

TABLE 1 — SAMPLE TEST RESULTS, 1960 STANDARD SPECIFICATION TESTS*

Specification Designation	AASHO Test Method	R-3655** Standard 40-50	R-3656** Standard 85-100	R-3657** Standard 120-150	R-3618** Golden Bear 85-100	AASHO*** 85-100
Flash Point PMCT °F	T73	510	455	465	495	556 (COC)
Penetration of Original Sample at 77°F	T49	39	96	119	85	91
Loss on Heating 5 Hrs. at 325°F, % Max. (Calif. Test Method No. 337 Thin Film Procedure)	-	.18	.47	.72	.14	0.03 (Loss on Heat)
Penetration After Loss at 325°F, % Original Penetration	T49	64	48	50	69	84
Ductility at 77°F cm After Loss at 325°F	T51	100+	100+	100+	100+	-
Penetration Ratio:						
$\frac{\text{Pen. } 39.2°\text{F - 200 gm - 1 min.}}{\text{Pen. } 77°\text{F - 100 gm - 5 sec.}}$	T49	24	32	32	26	-
Furol Viscosity at 275°F	T72	246	174	138	111	-
Heptane Xylene Equivalent, %	T102	15-20	20-25	15-20	0-5	Negative (Spot Test Standard Napthane)
Solubility in CCl_4, %	T45	99.97	99.97	99.96	99.95	99.91 (CS_2)

* Results for samples R-3655, R-3656, R-3657, and R-3618 obtained by Materials and Research Department, State of California, Division of Highways, and transmitted to University of California, October 29, 1963.

** Sample number.

*** Test results for AASHO material as reported in Table 44, Special Report 61B, Highway Research Board, 1962.

TABLE 2 — SAMPLE TEST RESULTS, NEW 1962 TENTATIVE SPECIFICATION TEST*

Specification Designation	Test Method	Specification for 85-100 Grade	R-3655** Standard 40-50	R-3656** Standard 85-100	R-3657** Standard 120-150	R-3618** Golden Bear 85-100	R-3722** AASHO Sample
Flash Point PMCT °F, Min.	AASHO T73	475	510	455	465	495	-
Penetration of Orig. Sample at 77°F	AASHO T49	85-100	39	96	119	85	90
Stain No. of Original Sample Max. After 120 Hrs., 140°F - 50 psi	ASTM D1328-58T	10	7.5	10	10	5.0	9.5
Viscosity cs on Original Sample:	ASTM D445						
140°F Minimum x 10^5		2.2	3.909	1.528	.902	1.048	1.283
225°F Minimum		1800	2841	1585	1374	1215	1633
325°F Minimum		200	130	99	87	61	102
Cohesiograph Reading:	Calif. 350						
Original, Minimum Inches		.80	.99	.78	.70	.61	.68
Gain 0-24 Hrs., Minimum Inches		.08	.17	.08	.08	.04	.12
Rolling Thin Film Test at 325°F, 75 Minutes	Calif. 346						
Pen. Residue, 77°F, Min.	AASHO T55	55	26	53	49	61	52
Ductility Residue, 77°F, Min.	AASHO T51	75	100+	100+	100+	100+	100+
Durability Test:	Calif. 347						
Viscosity of Residue After Durability Test:							
Megapoises at 77°F, Shear Rate 0.05 Sec.$^{-1}$ Maximum	Calif. 348	20	368	46	35	8.8	13
Megapoises at 77°F, Shear Rate 0.001 Sec.$^{-1}$ Maximum		60	980	230	129	8.8	39
Micro Ductility of Residue After Durability Test, 77°F, 1/2 cm/min Minimum mm	Calif. 349	10	2	2	4	74	7
Solubility in CCl_4, Orig. Sample, % Min.	AASHO T45	99	99.97	99.97	99.96	99.95	99.97
Rolling Thin Film Test at 375°F, 75 Minutes:	Calif. 346						
Penetration Residue 77°F, Minimum	AASHO T49	45	19	33	43	48	-
Ductility Residue 77°F, Minimum	AASHO T51	60	100+	76.5	100+	100+	-

* Results for all samples obtained by Materials and Research Department, State of California, Division of Highways and transmitted to University of California October 29, 1963.

** Sample number.

TABLE 3 — DESIGNATION OF ASPHALTS USED IN FATIGUE STUDIES

Sample	Materials & Research Designation	Pen. at 77°F Mat. & Res. Lab.	U.C. Designation	Pen. at 77°F U.C. Lab
Standard 40-50	R-3655	39	S-2	-
Standard 85-100	R-3656	96	S-1	90
Standard 120-150	R-3657	119	S-3	-
Golden Bear 85-100	R-3618	85	G	89
AASHO 85-100	R-3722	90	E	90
Shell 85-100*	-	-	A	89
Shell 40-50*	-	-	B	41

*These materials were not part of the investigation but results of specimens containing these materials are included in the report to illustrate certain behavior characteristics.

Results of these mix designs are shown in Figs. 2, 3 and 4. The asphalt content for specimens containing asphalts S-1 was selected at 5.9 percent and for asphalt G at 5.8 percent.

Beam specimens of approximately 2 x 3 x 12 in. were fabricated using the Triaxial Institute Kneading Compactor and the procedure described in Ref. 1.* After compaction and prior to testing, the top surface and both sides of each of the beams were sawed with a diamond tipped table saw to obtain a greater degree of uniformity between specimens. This sawing resulted in test specimens with final dimensions of 2.0 x 2.7 x 12 in. The number of specimens prepared and tested for each series is shown in Table 4. Also shown in this table are the number of specimens tested in the B series for purposes of comparison. It will be noted that air void content rather than density is indicated for the specimens. From our test results to date, and also as indicated by Saal and Pell[2] the percent air voids could appear to be significantly related to the fatigue life for a given mix. Thus to have some measure of the variability of results for comparison within a series and between series, mean values and standard deviations are shown.

A total of 160 specimens have been prepared to date using the project asphalts. In addition, the results presented utilizing the Shell materials are based on a total of approximately 200 specimens.

TABLE 4 — PRELIMINARY TEST RESULTS ON LABORATORY PREPARED SPECIMENS

Test Series	Tests at 75°F				Tests at 40°F			
	Total No. of Specimens	Percent Air Voids			Total No. of Specimens	Percent Air Voids		
		Mean	Standard Deviation	Coef. of Variation, Percent		Mean	Standard Deviation	Coef. of Variation, Percent
E	30	4.42	0.356	8.1	37	5.12	0.367	7.1
G	27	3.92	0.367	9.4	26	5.36	0.476	8.9
S-1	10	4.75	0.236	5.0	27	5.47	0.585	10.7
B	43	4.91	0.531	10.8	77	4.20	0.305	7.3

*Numbers refer to references appended to this report.

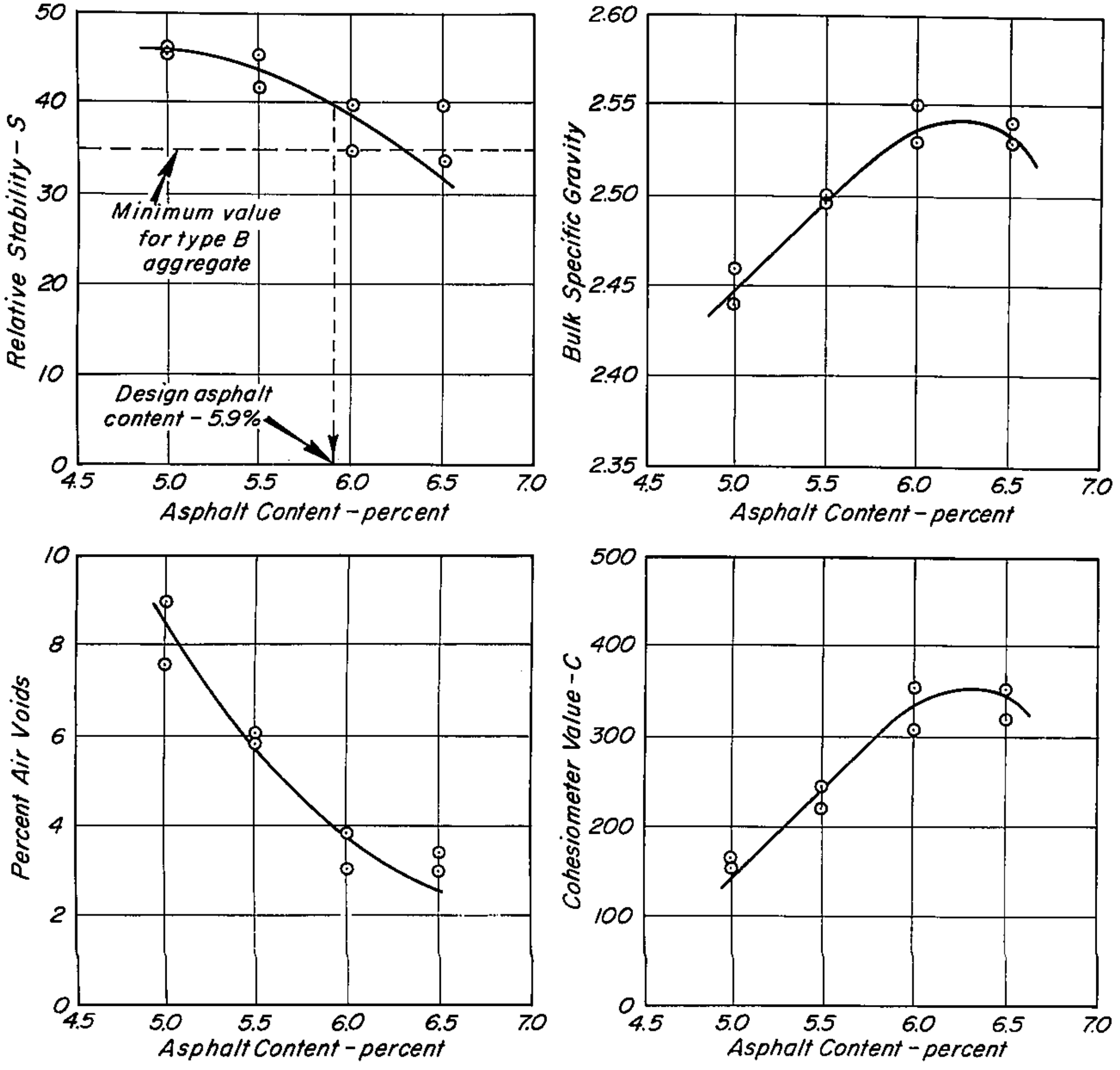

Fig. 2 — Summary of preliminary mix design data for specimens prepared with asphalt sample E.

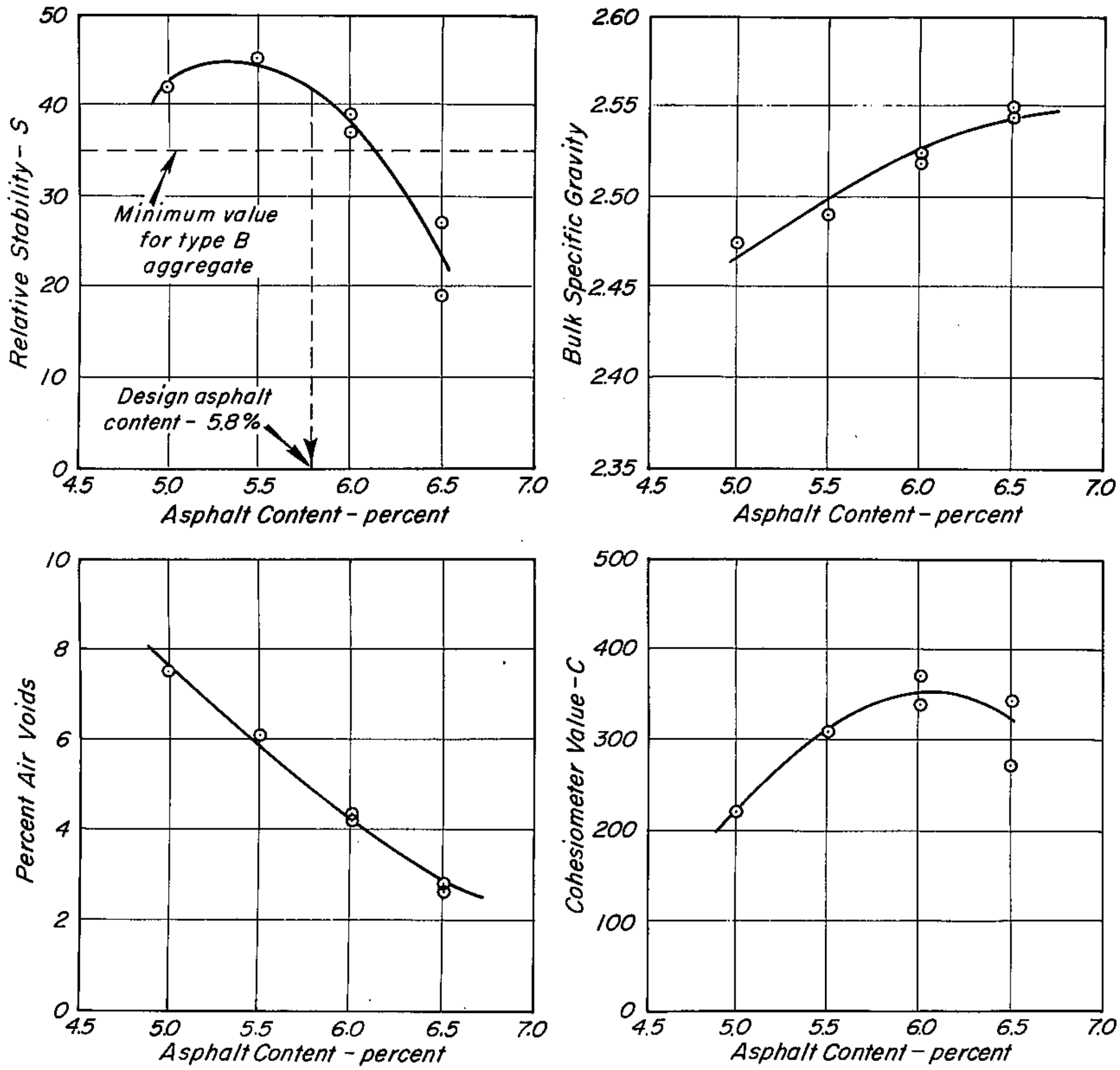

Fig. 3 — Summary of preliminary mix design data for specimens prepared with asphalt sample G.

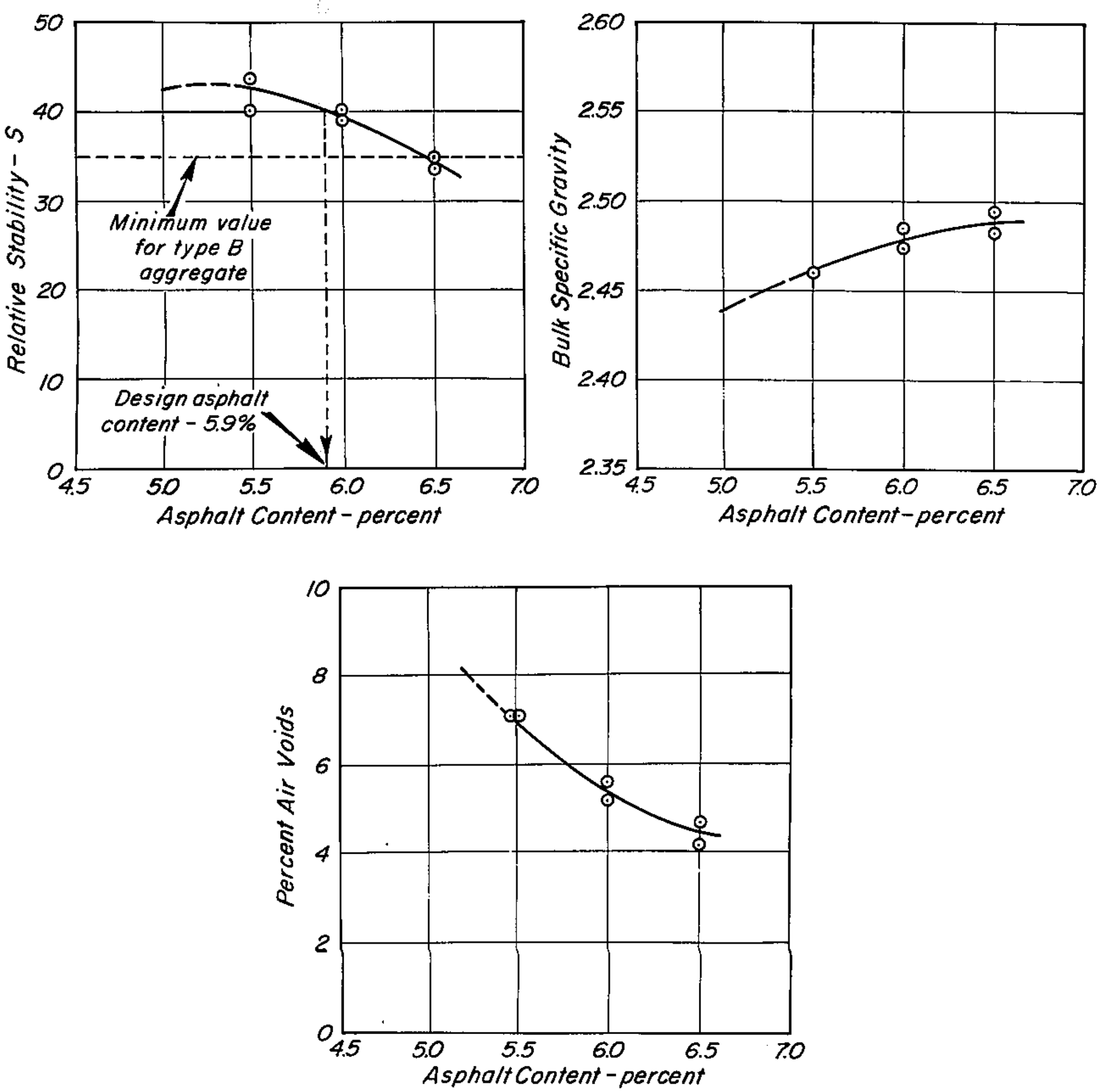

Fig. 4 — Summary of preliminary mix design data for specimens prepared with asphalt sample S-1.

TEST EQUIPMENT AND PROCEDURES

The equipment used to test the beam specimens in repeated flexure is illustrated schematically in Fig. 5a. Load is applied to the specimen resting on the spring base through an air cylinder acting against a loading yoke. The size of the cylinder is varied depending upon the range of loads to be applied to the specimens. Frequency and duration of load are controlled electronically and can be varied over a wide range of time from a minimum of 0.1 sec. A more detailed description of the operation of this equipment is included in Refs. 1 and 3.

For this investigation, one duration (approximately 0.1 sec) and one frequency (30 applications per min) of load application were utilized. It should be noted at this point that frequency of load application might appear to be an important variable and should be considered in future investigations. In previous work[3] it was reported that frequency had no effect on specimen behavior in repeated flexure in the range 3 to 30 repetitions per min. Recent preliminary investigations, however, in a frequency range from 30 to 100 repetitions per minute would tend to indicate a reduction in fatigue life at a particular stress or strain level with increase in frequency in this latter range.

A typical load and strain vs. time trace is illustrated in Fig. 5b. It will be noted that the strain has completely recovered prior to the application of the next load.

In fatigue testing, either controlled stress (or controlled load) or controlled strain (or controlled deflection) tests may be performed. For this investigation, constant strain amplitude testing was selected with the intensity of tensile strain during the test monitored by SR-4 type strain gages nominally 1 in. in length bonded to the lower surface of each of the beams with a thin film of epoxy cement.

Two gages were used on each beam directly below the center of load application in the region of maximum strain intensity. The longitudinal axis of each gage was located 1/4 in. from the edge of the beam and parallel to it. During the test, a layer of teflon tape was used between the gage and the flexible diaphragm of the spring base to minimize friction between the gage and the base.

To determine the fatigue life at a particular strain level, a series of specimens were subjected to varying numbers of load applications. At the end of a particular number of applications, the specimen under investigation was tested in simple flexure using third point loading and the modulus of rupture was computed from the maximum observed load. By expressing this result as a percentage of the modulus determined for a series of unflexed specimens, an indication of the development of damage with load repetitions is obtained. Typical data of this type are illustrated in Figs. 6 and 7 for specimens prepared with asphalt sample E. Table 5 contains a summary of the moduli for the unflexed specimens for all series both at 75°F and 40°F. For this investigation the fatigue life (cycles to failure) is defined as the number of load applications to cause a 10% reduction in the modulus of rupture. This definition is illustrated in both Figs. 6 and 7.

By performing tests of this type for a series of strain levels at one temperature, a strain vs. cycles-to-failure curve such as that illustrated in Fig. 8 can be obtained. By repeating this process at different temperatures a family of curves can be obtained, two of which are illustrated on Fig. 8, that is, one at 40°F and the other at 75°F.

TABLE 5 — MODULUS OF RUPTURE OF UNFLEXED SPECIMENS

Test Series	Tests at 75°F				Tests at 40°F			
	Number of Specimens	Mean Modulus, psi	Standard Deviation, psi	Coef. of Variation, Percent	Number of Specimens	Mean Modulus, psi	Standard Deviation, psi	Coef. of Variation, Percent
E	12	102	10.3	10.0	14	553	39.1	7.1
G	12	80.8	10.8	13.4	9	936	88.6	9.5
S-1	4	110	3.0	2.7	10	690	67.0	9.7
B	20	146	27.3	18.7	29	940	149.7	15.9

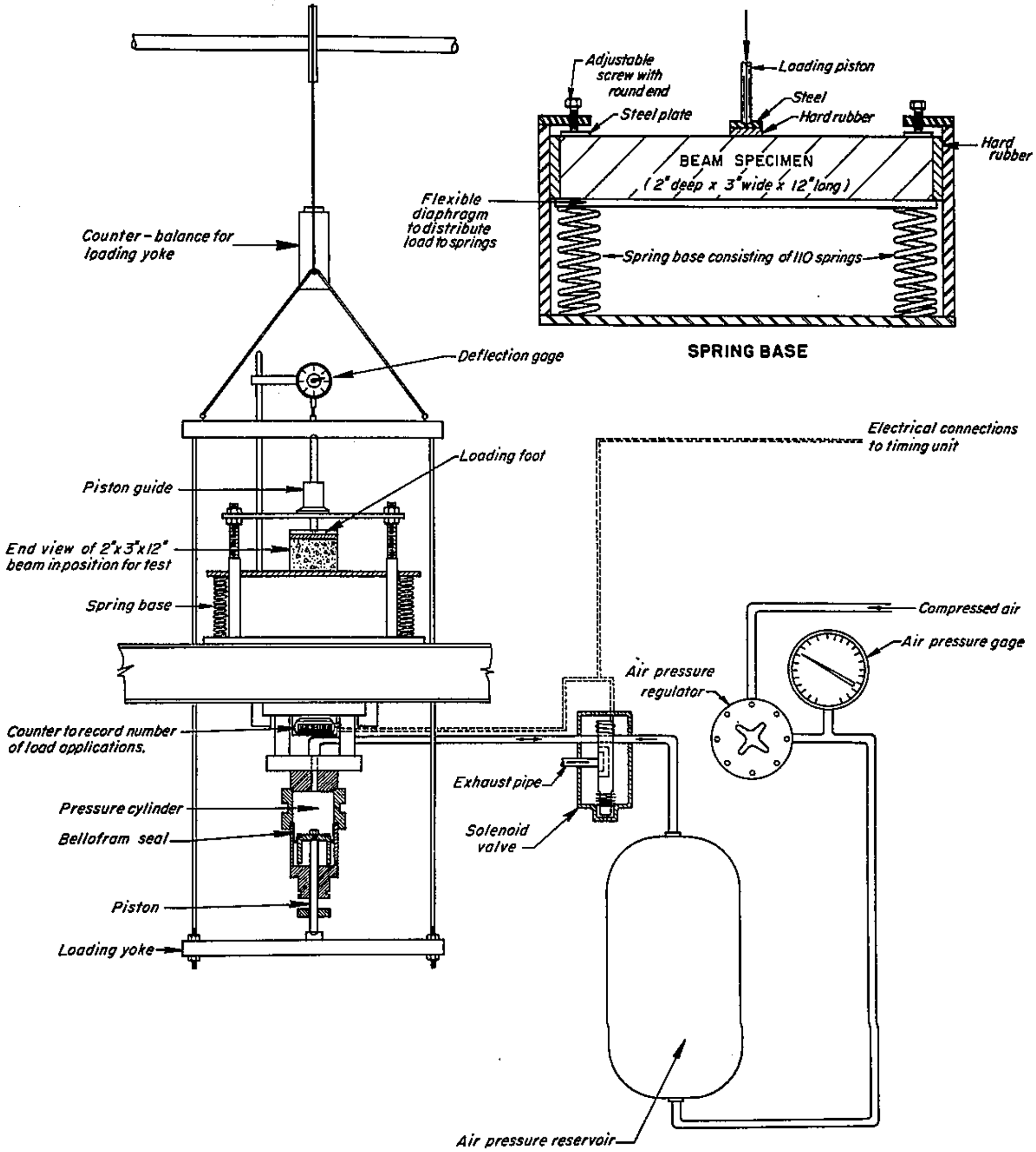

Fig. 5a — Equipment for testing beam specimens in repeated flexure.

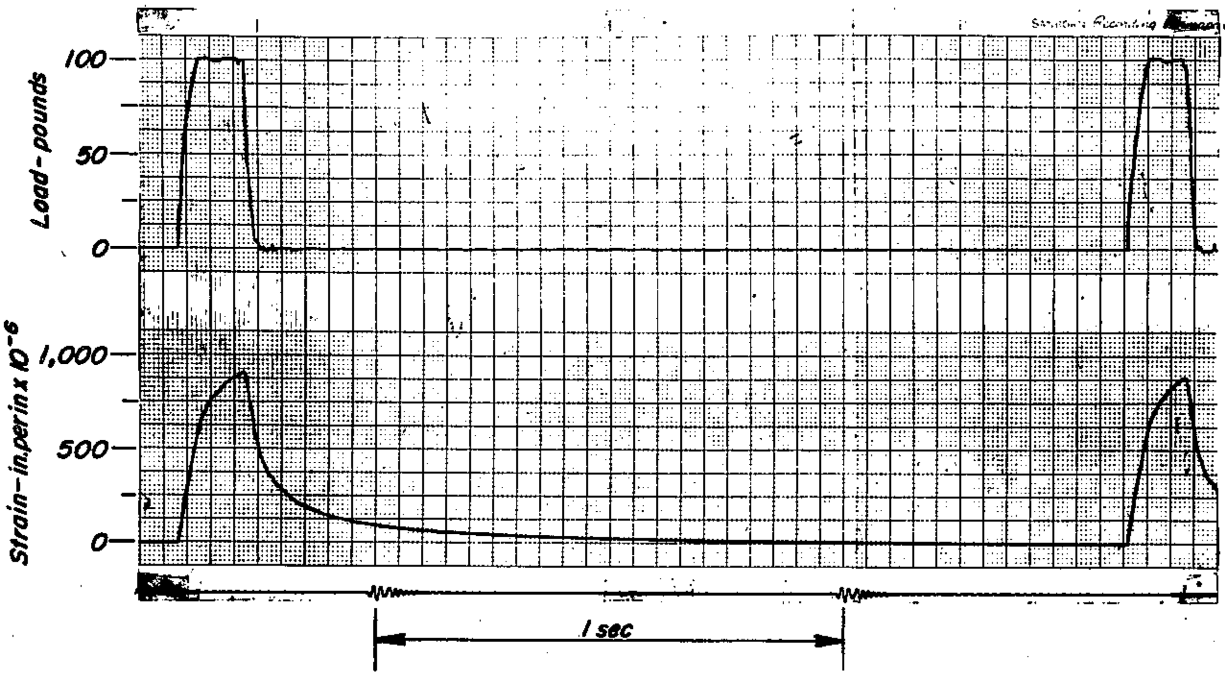

Fig. 5b — Typical load vs. time and strain vs. time recordings asphalt sample E, 900×10^{-6} in. per in. at 75°F, 400,000 load applications.

In this discussion it has been emphasized that the results were obtained from constant-strain-amplitude, repeated-load tests. During the course of testing of an individual beam it may or may not be necessary to vary the load to maintain constant strain. Figs. 9 and 10 illustrate the variations of load with number of load applications for specimens prepared with asphalt sample E.

Fig. 9 illustrates data generally typical of those obtained for all specimens at 75°F; the load remains essentially constant to the estimated fatigue life. At 40°F, on the other hand, the load must gradually be reduced to maintain a constant strain amplitude; this point is illustrated in Fig. 10.

Above it was stated that two strain gages are used on each beam specimen. The average of the strain values obtained from both gages is that reported, since it would be fortuitous if both gages indicated exactly the same strain, even near the start of a particular test. Figs. 10 and 11 are presented to illustrate the extent of variation of the actual strain readings from the mean value reported.

In Fig. 11, for specimens prepared with asphalt sample E and tested at 75°F at a mean strain level of 900×10^{-6} in. per in., it will be noted that the deviation from the mean is less than 5 percent in the majority of specimens tested; that is, the strain in the two gages varied at the most from 855 to 945×10^{-6} in. per in. Also it will be noted that this difference remained essentially constant for the range of load applications investigated. At 40°F, on the other hand, a different behavior pattern was obtained, as illustrated in Fig. 12. Initially the deviation from the mean was of the same order as that noted at 75°F. However, with increasing numbers of load applications, the difference in strain between the gages increased; also the load, as indicated earlier, decreased. This behavior is quite interesting in that it would indicate a difference in damage progression (crack development), which is temperature dependent. At 75°F, for example, it would appear that a number of small cracks, randomly distributed, in the vicinity of the section subjected to maximum strain gradually develops with number of load applications. At 40°F, however, the data would seem to indicate that one large crack generally develops* as

* Pell[4,5] has presented interesting data on crack propagation recently which tend to substantiate this hypothesis.

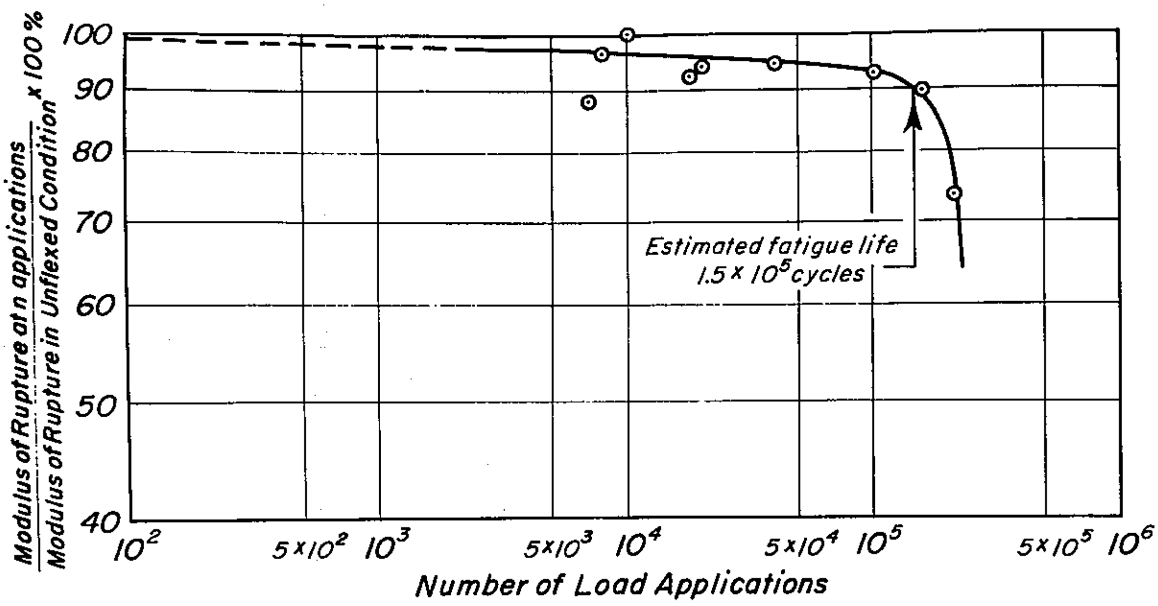

Fig. 6 — Modulus of rupture vs. number of load applications for specimens prepared with asphalt sample E at strain amplitude of 900 x 10^{-6} in. per in. at 75°F.

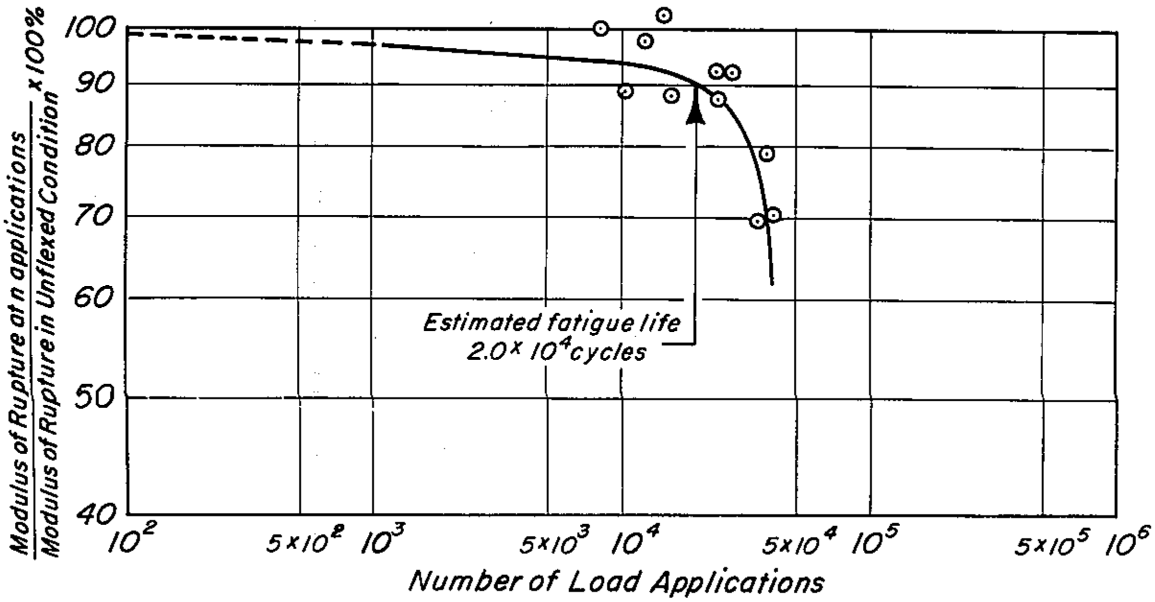

Fig. 7 — Modulus of rupture vs. number of load applications for specimens prepared with asphalt sample E at strain amplitude of 900 x 10^{-6} in. per in. at 40°F.

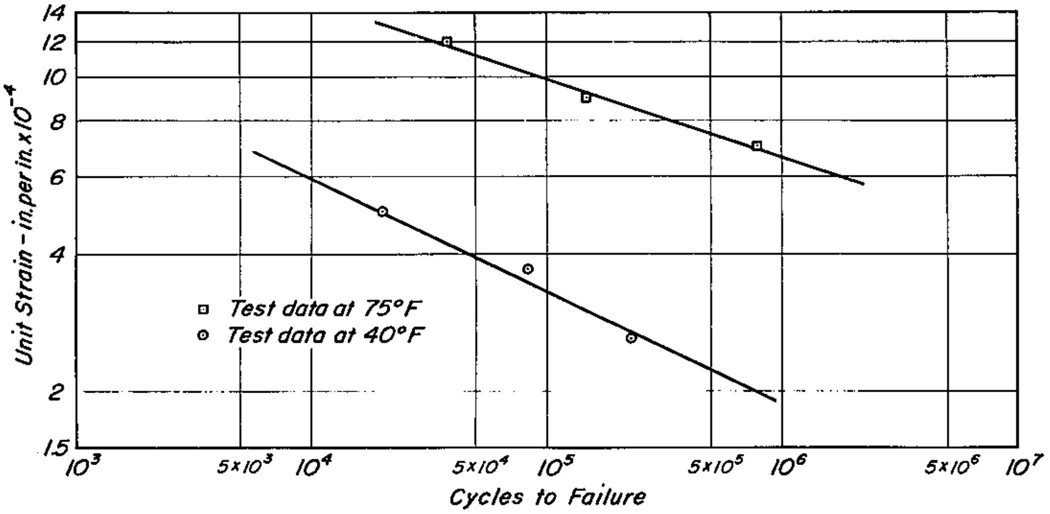

Fig. 8 — Results of constant strain amplitude fatigue tests at 40°F and 75°F for specimens prepared with asphalt sample E.

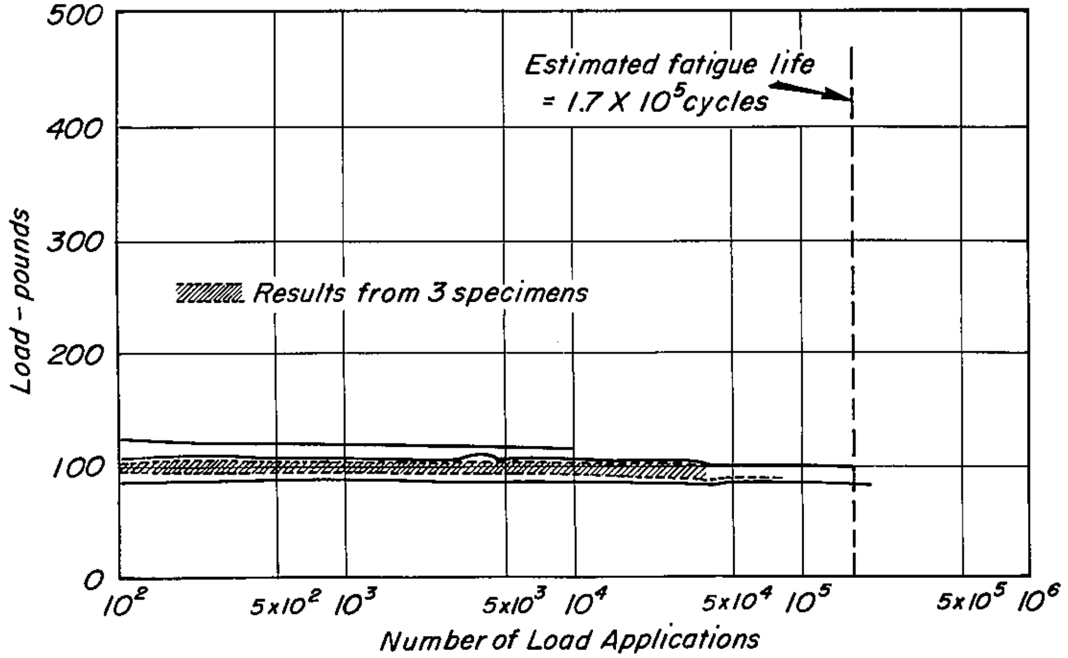

Fig. 9 — Load required to produce 900 x 10^{-6} in. per in. strain vs. number of load applications for specimens prepared with asphalt sample E at 75°F.

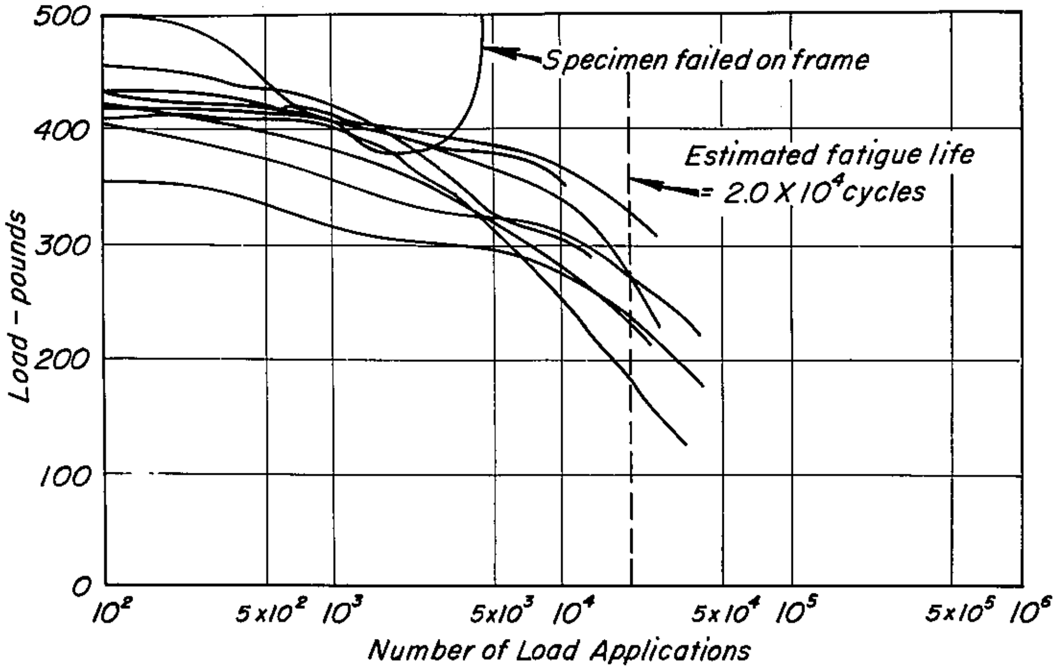

Fig. 10 — Load required to produce 500 x 10^{-6} in. per in. strain vs. number of load applications for specimens prepared with asphalt sample E at 40°F.

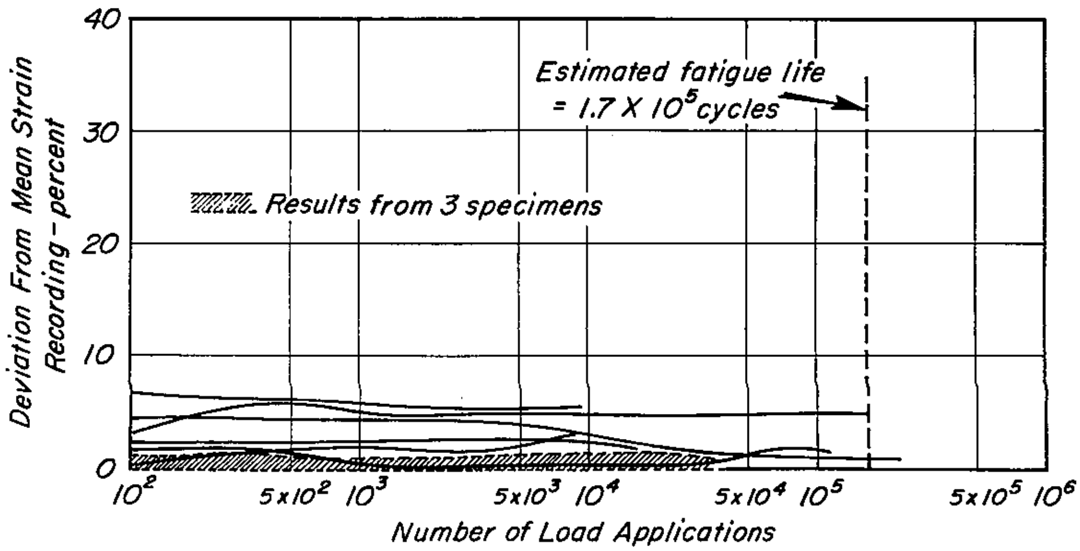

Fig. 11 — Percent deviation from mean strain recording vs. number of load applications for specimens prepared with asphalt sample E — strain level of 900 x 10^{-6} in. per in. at 75°F.

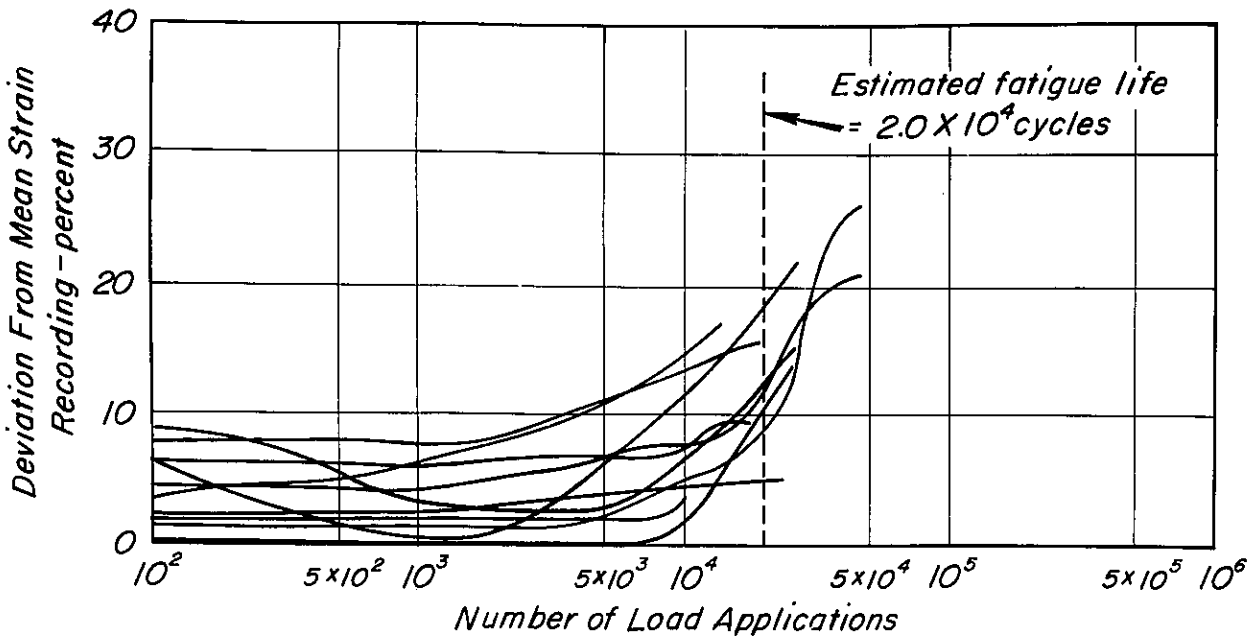

Fig. 12 — Percent deviation from mean strain recording vs. number of load applications for specimens prepared with asphalt sample E — strain level of 500 x 10^{-6} in. per in. at 40°F.

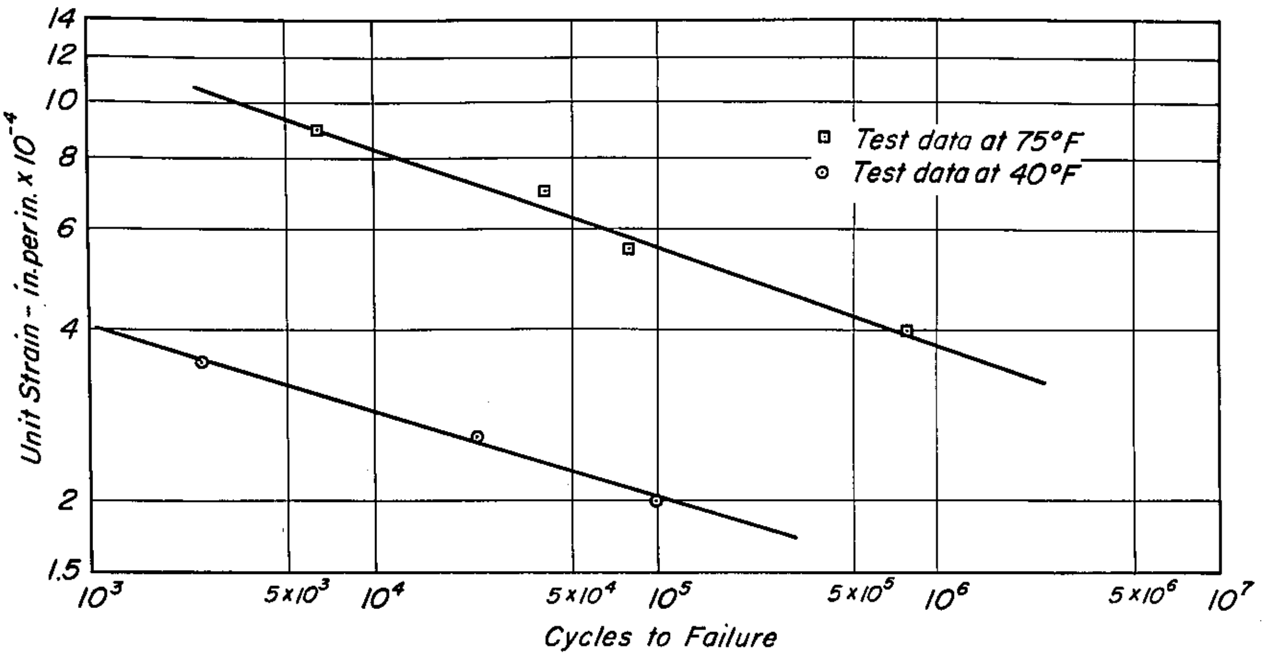

Fig. 13 — Results of constant strain amplitude fatigue tests at 40°F and 75°F for specimens prepared with asphalt sample G.

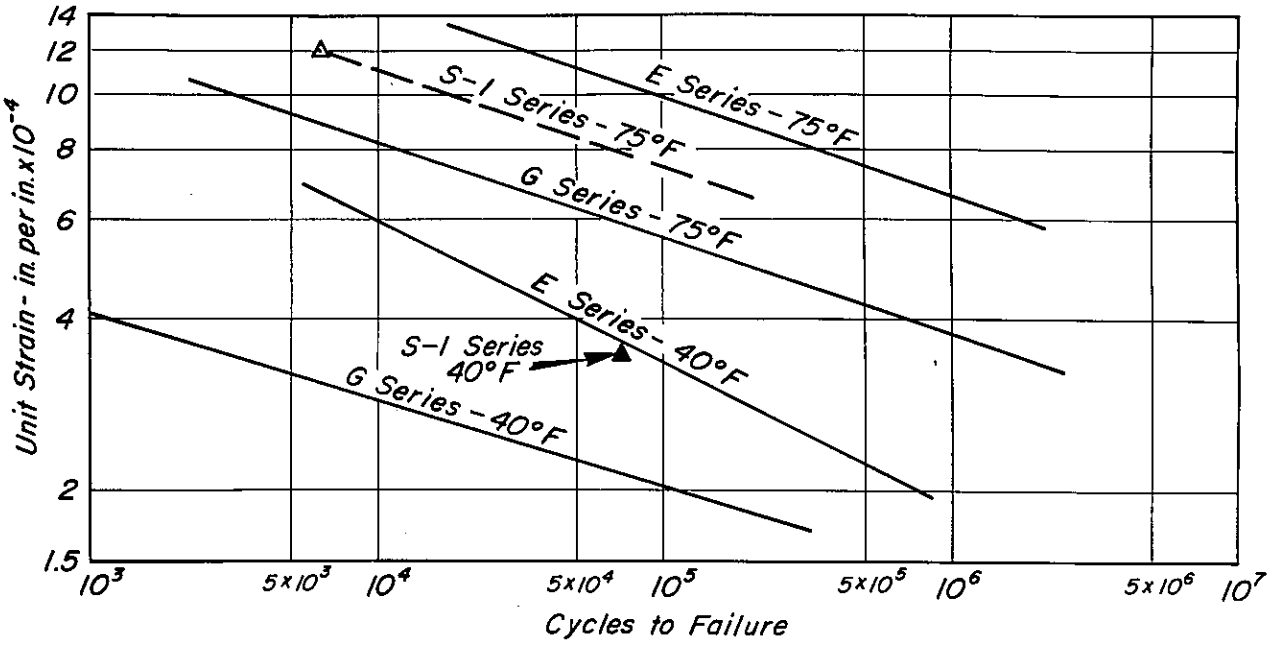

Fig. 14 — Summary of fatigue test data on specimens containing Watsonville aggregate and project asphalts.

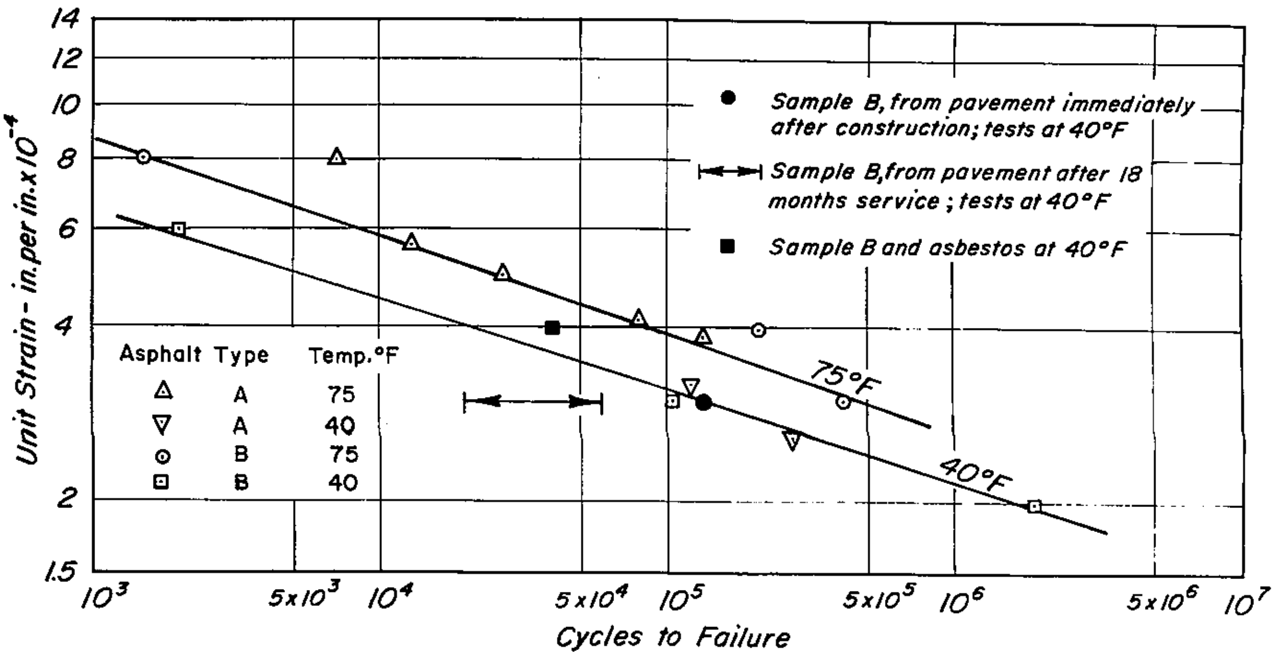

Fig. 15 — Results of constant strain amplitude tests on materials used in Shell Ave. test road — Contra Costa County.

indicated by the difference in strain readings between the two gages and that this propagates relatively quickly near the estimated fatigue life as indicated by the large rate of increase in deviation of strain illustrated in Fig. 12. Greater attention should probably be directed to this aspect in future investigations in order to better define this facet of the problem.

FATIGUE DATA

Utilizing the tests and analyses described in the previous section, strain vs. cycles-to-failure data are presented in Figs. 8, 13, and 14 for specimens containing the project asphalts and the Watsonville granite at temperatures of 40°F and 75°F. Figure 15 contains data obtained from specimens using asphalt samples A and B and a crushed basalt aggregate from Contra Costa County. This data is presented to illustrate certain aspects of behavior which will also be demonstrated with project asphalts and the Watsonville granite during this next year.

In examining the detailed results presented in Figs. 8 and 13 for asphalt samples E and G, respectively, it will be noted that there is a considerable difference in the fatigue lives at 75°F and 40°F, approximately two orders of magnitude at a particular strain level.

The results of the E and G series are summarized in Fig. 14 for purposes of comparison. Also shown in the figure are the results of tests at one strain amplitude at 75°F and 40°F for specimens prepared with sample S-1. It will be noted that the curves of strain vs. cycles-to-failure for asphalts E and G are parallel at 75°F. For this reason the curve for asphalt S-1 is drawn through the one test point parallel to these curves as a tentative indication.

At 40°F, the specimens prepared with sample S-1 exhibit the same behavior as specimens prepared with asphalt E at the one strain level tested.

From a cursory examination of the curves, one might conclude that asphalt sample E would appear to give better performance in fatigue than asphalt sample G; also, tentatively, it would appear that asphalt sample S-1 would appear to perform the same as asphalt sample E at 40°F. Before these conclusions are drawn, however, one should examine the laboratory data in more detail. This point will be discussed further in the next section.

Figure 15 illustrates laboratory data of the same type as that shown in Figs. 8, 13, and 14. In this case, however, the specimens were prepared with a crushed basalt conforming to the 3/4-in. maximum-medium gradation of the 1960 State of California Specifications. As noted earlier, asphalt A is an 85-100 penetration asphalt cement and asphalt B is a 40-50 penetration material, both of which meet the 1960 specification requirements. This data is presented parimarily to illustrate the point that the hardness of the asphalt appears to have no effect on the relationship between strain and cycles to failure at different temperatures. It will be noted in this plot that 85-100 and 40-50 materials produce the same relationship both at 75°F and 40°F. This may in part be due to the fact that the difference in mix properties between 75°F and 40°F is very large in comparison to the difference between the 40-50 and 85-100 materials at either temperature.

Also shown in this figure, primarily for informational purposes since the data is somewhat limited, are the results of laboratory tests performed on specimens sawed from the pavement containing the 40-50 penetration asphalt in the field, in this case, Shell Ave. in Martinez, California. Tests performed on the specimens taken immediately after construction exhibit the same results at 40°F as the laboratory prepared specimens. On the other hand, it would appear that some damage has been induced in the pavement sampled 18 months after construction since the range of values obtained for a series of specimens lies to the left of the 40°F line as shown. This could be due in part to embrittlement of the asphalt and in part due to traffic induced damage.

Again, for informational purposes, some preliminary results of tests at 40°F using the mix with asphalt sample B and 2.5% by weight of asbestos fibers are illustrated in Fig. 15. It will be noted that the results of the one series lie somewhat to the right of the 40°F line for the conventional mixes. This type of data has also been obtained by another investigator for the same mixtures.

DISCUSSION OF FATIGUE TEST DATA

As noted in the previous section, by examining Fig. 14 one might tend to rate the performance of the asphalts at 75°F in the order asphalt E, asphalt S-1, and asphalt G. Similarly at 40°F, the order would appear to be asphalt E and S-1 together followed by asphalt G. Before such conclusion are drawn, however, the testing procedure again should be noted. The tests are conducted at constant bending strain amplitude. To obtain this strain level, the load is adjusted to different intensities, depending on the temperature and the asphalt being used. As an example, in the following table the loads required to produce 350×10^{-6} in. per in. strain at 40°F are shown for the various test series.

TABLE 6 — LOAD TO PRODUCE STRAIN AMPLITUDE OF 350×10^{-6} IN. PER IN. AT 40°F

Test Series	Start of Test		At Estimated Fatigue Life Average - lb
	Average - lb	Range - lb	
E	295	270-345	205
G	490	365-540	380
S-1	387	310-460	236

It can be seen that considerably greater load is required to produce the same strain in specimens prepared with asphalt G as compared with those prepared using asphalt E. Specimens containing asphalt S-1 required an intermediate load.

This data would suggest that consideration also be given to stress vs. cycles-to-failure data and stiffness characteristics for the specimens containing the various asphalts. With the particular apparatus, however, it is difficult to estimate the stress since the loading pattern on the beams is complicated by the spring base.* The data noted in Table 6 were obtained for one strain level at 40°F. At 75°F a similar trend is noted in that the loads required to produce 1200×10^{-6} in. per in. strain are 170 lb, 155 lb, and 145 lb, respectively, for specimens produced with asphalts G, S-1 and E in that order.

To attempt to place the data obtained thus far in proper perspective, it would seem appropriate to consider overall pavement behavior. One approach is to utilize layered system theory[6] to assist in this examination. To do this, as noted below, stiffnesses or some indication of the moduli of the pavement materials are required under the particular conditions of loading. In a qualitative manner one might conclude that when the asphalt layer is relatively thin in comparison to the overall pavement structure, the asphalt mixture which exhibits a longer fatigue life at a given strain level would perform more satisfactorily in the field; the reason being that the strain which develops in the asphalt layer will be controlled by the underlying materials primarily. In thicker asphalt layers, however, one cannot generalize since the strain which develops will be a function both of the thickness of the layer and the relative stiffness of the asphalt layer and the underlying material.

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*The equation for the center moment for a beam of finite length on an elastic foundation subjected to center loading is:

$$M = \frac{P}{4\beta} \cdot \frac{\cos h\beta\ell - \cos \beta\ell}{\sin h\beta\ell + \sin \beta\ell}$$

where:

$$\beta = \sqrt[4]{\frac{k}{EI}}$$

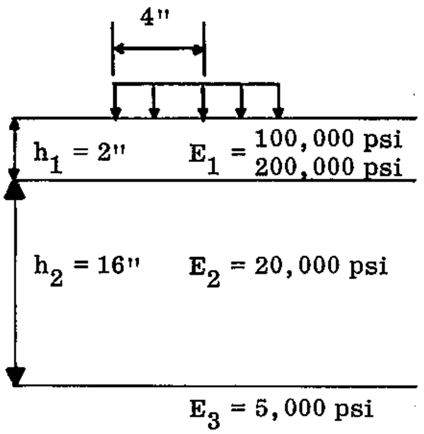
ℓ = length of beam

k = subgrade modulus

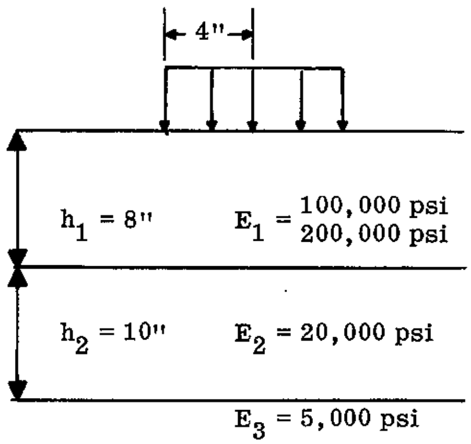
Thus it can be seen that it is relatively difficult to obtain quantitative information on the actual stresses without additional measurements of E (or stiffness) under actual loading conditions including the effects of time and temperature.

Utilizing three-layer theory for the following conditions, some relative indications of the above statements can be obtained

Case 1



Case 2



The thin layer will be represented by 2 in. of asphalt concrete and the thick layer by 8 in. of the same material. Further, it will be assumed that the modulus or stiffness of the asphalt layer will vary by a factor of two, i.e., 100,000 psi and 200,000 psi, respectively. For these conditions and utilizing three-layer elastic theory, the tensile strain on the underside of the asphalt layer can be determined. To obtain a relative indication of the effect of stiffness and layer thickness, consider the strain computed for the case h_1 = 2 in. and E_1 = 100,000 as unity. For the same condition, when the stiffness is increased to 200,000 psi, the strain on the underside of the asphalt layer is 84 percent of the value obtained for the case with E_1 = 100,000 psi.

When h_1 is increased to 8 in. (case 2), the strains are 38% when E_1 is 100,000 psi and 27% when E_1 is 200,000 psi, respectively, of the value obtained for h_1 = 2 in. and E_1 = 100,000 psi. These computations are summarized in Table 7.

Also included in the table is the ratio of the strains for both cases 1 and 2. It will be noted that the effect of increase in pavement stiffness increases as the pavement thickness increases.

TABLE 7 — RELATIVE TENSILE STRAIN AT UNDERSIDE OF SURFACE LAYER

	E_1 - psi		Strain Ratio
	100,000	200,000	
Case 1	1	.84	1.2
Case 2	.38	.27	1.4

Assume that the pavement with E_1 = 100,000 psi is represented by the mix prepared with asphalt sample E and the pavement with E_1 = 200,000 psi by the mix containing asphalt G. Justification for this comparison could be obtained, it would appear, from Table 6. Assume further that a 2-in. thick layer with asphalt E (E_1 = 100,000 psi) is subjected to a loading condition such that the resulting tensile strain on the underside of the layer is 600×10^{-6} in. per in. If the data shown in Fig. 14 are representative of field performance, then a service life of 10,000 cycles would be obtained. Correspondingly, for a 2-in. pavement prepared with asphalt G (E_1 = 200,000 psi) the strain would be about 500×10^{-6} in. per in. ($0.84 \times 600 \times 10^{-6}$ as indicated in Table 7). At this strain level the service life as obtained from Fig. 14 for asphalt G would be less than 1000 cycles. Hence for this condition, mixtures prepared with asphalt E would appear to give better performance.

For the 8-in. pavement, on the other hand, and for the condition of load which produced 600×10^{-6} in. per in. strain in the 2-in. layer (E_1 = 100,000 psi), the corresponding value of strain is about 230×10^{-6} in. per in. ($0.38 \times 600 \times 10^{-6}$ as indicated in Table 7). At this strain level, the service life for mixes containing asphalt E is approximately 400,000 cycles. In the same thickness of pavement, using asphalt G (E_1 = 200,000 psi), the strain level according to Table 7 would be about 160×10^{-6} in. per in. ($0.27 \times 600 \times 10^{-6}$ in. per in.) and the corresponding service life from Fig. 14 is 500,000 cycles.

At 75°F, using a base strain of 1200×10^{-6} in. per in. and the analysis explained in the above paragraphs, mixtures containing asphalt G do not, even at low strain levels (case 2), exceed the service lives of those containing asphalt E. However, while there is a significant difference in service lives at high strain levels there is virtually little difference at the lower level corresponding to the thicker sections of asphalt concrete.

At this point it should be emphasized that no attempt is being made to justify the California asphalt as compared to the mid-continent material. Rather the purpose here is to point out that comparisons of different materials is not a simple task.

The data presented in Fig. 15 are quite significant. In this case, comparing two asphalts from the same source, one obtains the same relationship between cycles-to-failure and strain at both 40°F and 75°F. It is possible that specimens prepared with the 40-50 material may be somewhat stiffer than those prepared with the 85-100 asphalt. Hence some increase in service life may be obtainable for specimens containing the harder material. The position of the 75°F curve for these materials is somewhat to the left of the data for asphalts contained in the project test series. However, this difference may be caused by aggregate gradation and/or aggregate type since it is presumed that the 85-100 pen. asphalt (A series) may correspond somewhat to asphalt S-1.

Fatigue tests performed on specimens taken from the section of pavement containing the 40-50 pen. asphalt shortly after construction appear to give essentially the same results as the laboratory specimens, when densities are the same. This is indicated by the one set of data at 300×10^{-6} in. per in. at 40°F shown in Fig. 15. It is interesting to note, however, that some of the field specimens existed at densities lower than those obtained for the laboratory specimens. Their corresponding fatigue lives were less. This point is also now being well demonstrated in constant stress amplitude tests in another investigation being conducted at the University laboratory.

For field specimens taken 18 months after construction, the fatigue life at 300×10^{-6} in. per in. strain is less than that obtained immediately after construction. However, the load to produce this strain was approximately 10% higher (considering average results) for the aged materials than for the unaged mixtures. It is interesting to note that the range in loads to produce this strain was quite large for this latter series indicating the possibility of some damage actually being done in the field; hence the range in values in Fig. 15 rather than a specific point.

In general the data obtained to date would suggest that:

1. Asphalt type would appear to effect the fatigue results in constant-strain-amplitude tests.
2. For asphalts from the same source, the initial hardness would not appear to affect the strain vs. cycles-to-failure relationship in constant-strain-amplitude tests at a particular temperature.
3. The density of specimens (and indirectly the air void content) would appear to affect test results with longer life associated with higher densities (lower air void contents). This relationship may not be a direct proportion, however, since at higher void contents the possibilities of stress concentrations exist.
4. To interpret the results of constant-strain-fatigue tests, the stiffness of the material under the particular conditions of loading as well as the strain vs. cycles-to-failure relationship would appear to be necessary.
5. Aggregate type and grading may affect the strain vs. cycles-to-failure relationship.

RECOMMENDATIONS FOR RESEARCH FOR 1963-64 AND BEYOND

As noted in the introduction, it was planned that during the year 1962-63 five asphalts and two different aggregates would be investigated. Approximately one-half the work has actually been completed. Thus, the first part of the program this next year will be to complete the original program which includes testing of the S-series asphalts, that is, the 40-50 and 120-150 materials as well as the 85-100 asphalt with the Watsonville aggregate. In addition, it will be necessary to perform a similar series of tests with a crushed gravel supplied by the Materials and Research Department. It is planned that the tests on the crushed gravel will not be as extensive as those performed with the granite since any trends obtained for this material should be similar to those obtained for the granite. This program may require the year to complete.

Beyond this, it is suggested that some consideration be given to the effect of density (and permeability) for a particular mix to demonstrate from a fatigue standpoint the importance of proper compaction in the field. Preliminary evidence would indicate this to be an important variable.

An important question which has arisen this year relates to the type of test being used to measure fatigue behavior. For the mixes whose performance is illustrated in Fig. 15, another investigator developed relationships between strain and cycles-to-failure based on constant stress tests which differ somewhat from those presented in this report (these latter data are also based on a different criterion for failure). Thus it would seem quite important to relate the behavior of mixes in both constant strain and constant-stress-amplitude tests. Beyond this, of course, it will be necessary to relate either or both of these laboratory test conditions to those which exist on an actual pavement. As noted earlier it is possible that the results of both types of tests may be required depending upon the usage of the pavement in the field. Finally, the possibility of an energy criterion rather than either stress or strain should be considered for the definition of fatigue failure.

During this next year it would also appear worthwhile to begin the planning of some field test sections specifically to examine fatigue cracking. This type of approach should permit the comparison of laboratory results with actual performance and should also permit the study of the effects of weathering on fatigue behavior. Some experience has been gained in this type of approach with the assistance of the County of Contra Costa on the Shell Ave. test road. Unfortunately, this particular road did not have as its specific goal the above-mentioned approach.

Along with the tie-in with field performance, some consideration should be given to cumulative damage aspects. Furtunately some preliminary work along these lines is being done in another investigation, the results of which should be available next June and should be of assistance in future planning for this project.

During this next year it is also hoped that some of the mixtures tested in the University laboratory can be examined by the Materials and Research Department. Specifically, it would appear quite important to examine the properties of the asphalts recovered from the beam specimens as compared to the properties of the original asphalts. If this suggestion is agreeable to the Department, specimens which have been subjected to loading will be transmitted to Sacramento. This phase of the investigation would appear to be important in that it should assist in taking the studies from a purely phenomenological level to one closer to explaining why fatigue actually occurs.

TEMPERATURE STRESS STUDY

As noted in the original proposal it was anticipated that some studies might possibly be conducted of the development of thermal stresses in asphalt concrete due to the restraint of temperature deformations.

During the summer of 1963, some progress was made along these lines using the project asphalts. Beam specimens were fabricated utilizing the same procedure as that used for the fatigue specimens. After compaction, however, each of the 2 in. x 3 in. x 12 in. specimens was sawed into four 1-in. x 1 in. x 12 in. specimens.

Each of these specimens was then placed in an invar frame shown schematically in Fig. 16. This apparatus is comparatively rigid and is thus capable of practically restraining any defor-

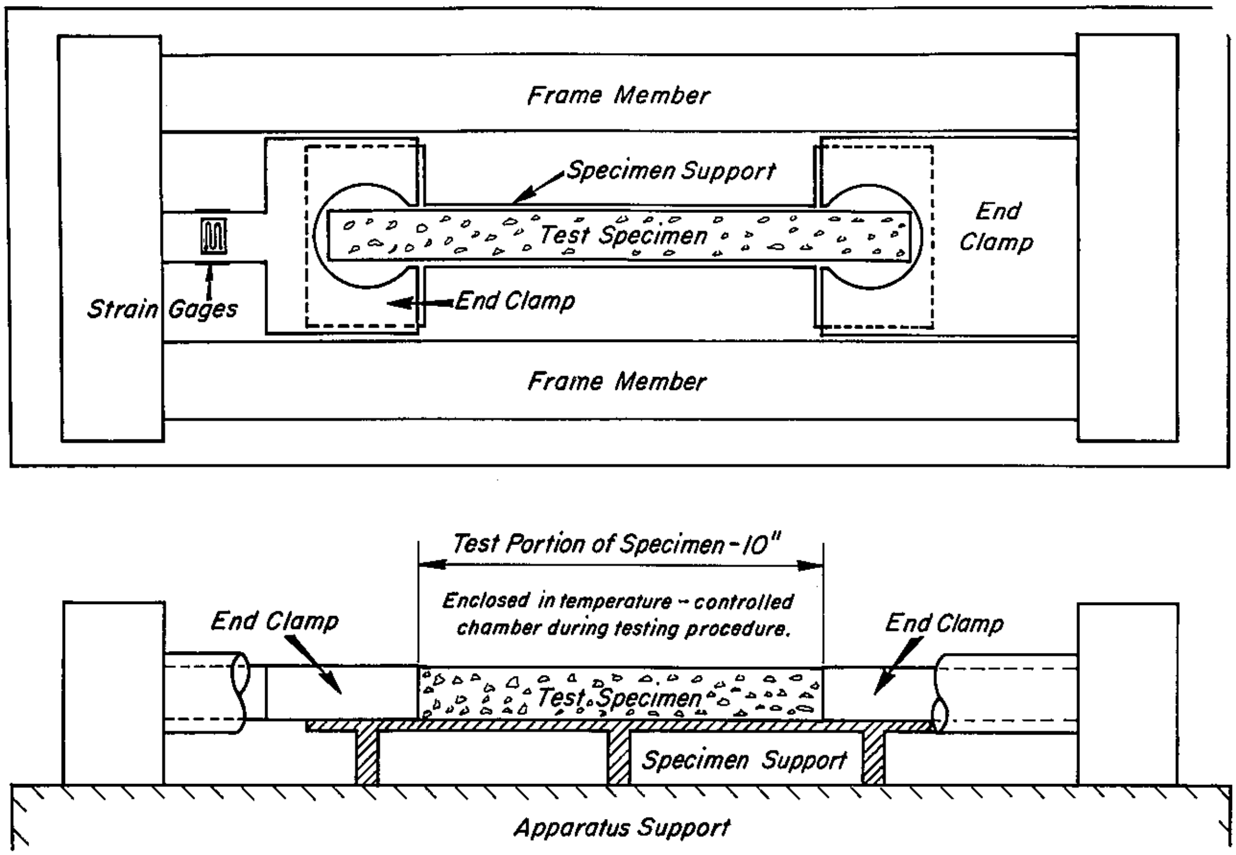


Fig. 16 — Thermal stress apparatus.

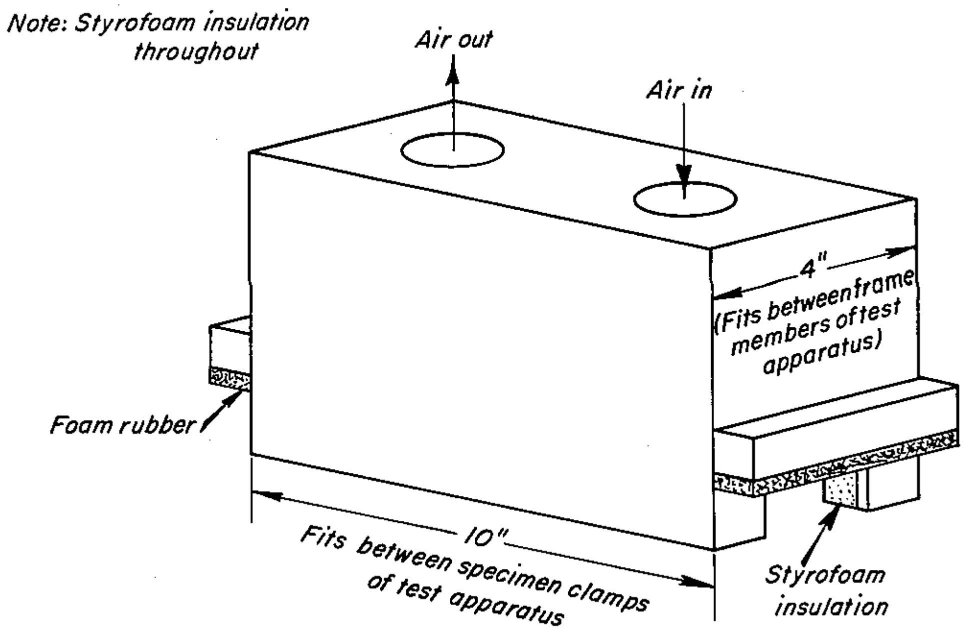


Fig. 17 — Controlled temperature cabinet for thermal stress experiments.

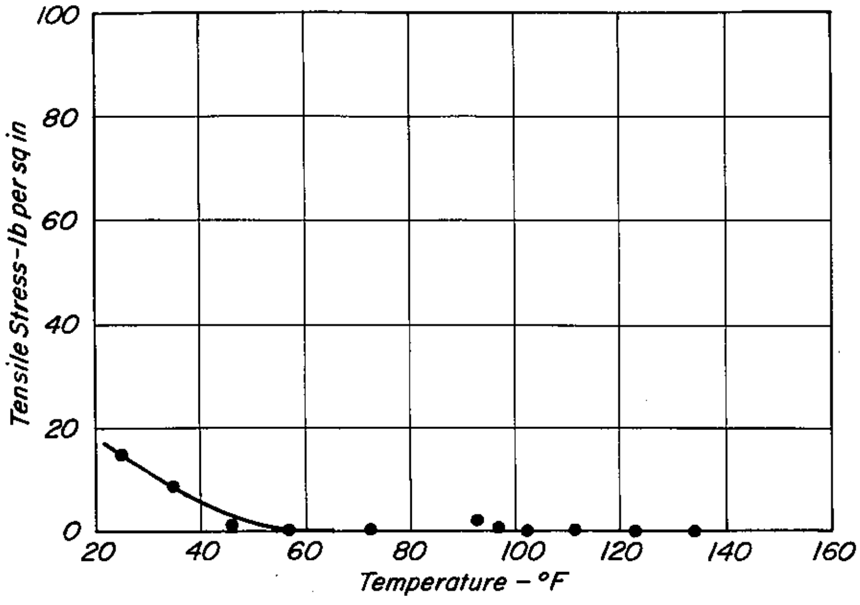


Fig. 18 -- Development of tensile stress in specimen containing asphalt sample E for rate of cooling of 11°F per hour.

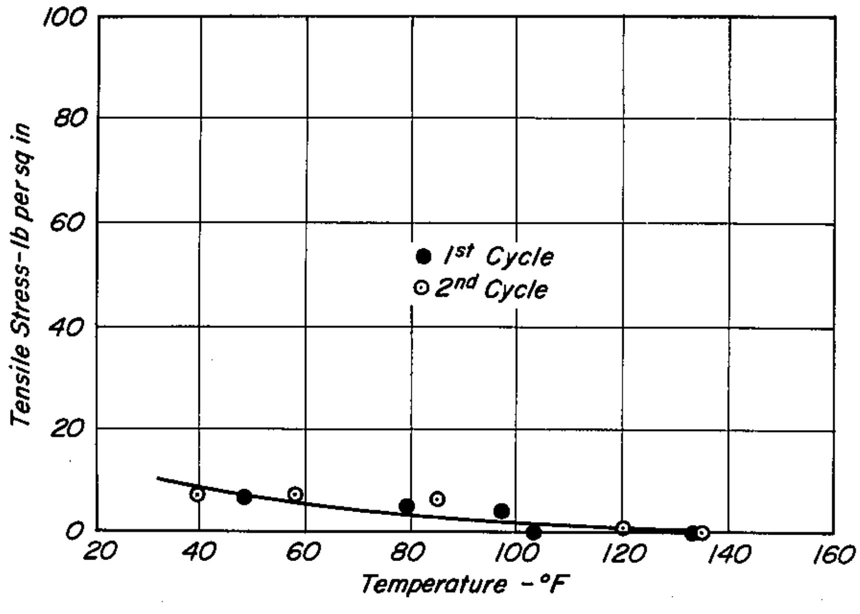


Fig. 19 -- Development of tensile stress in specimen containing asphalt sample G for rate of cooling of 14°F per hour -- 2 cycles of temperature change.

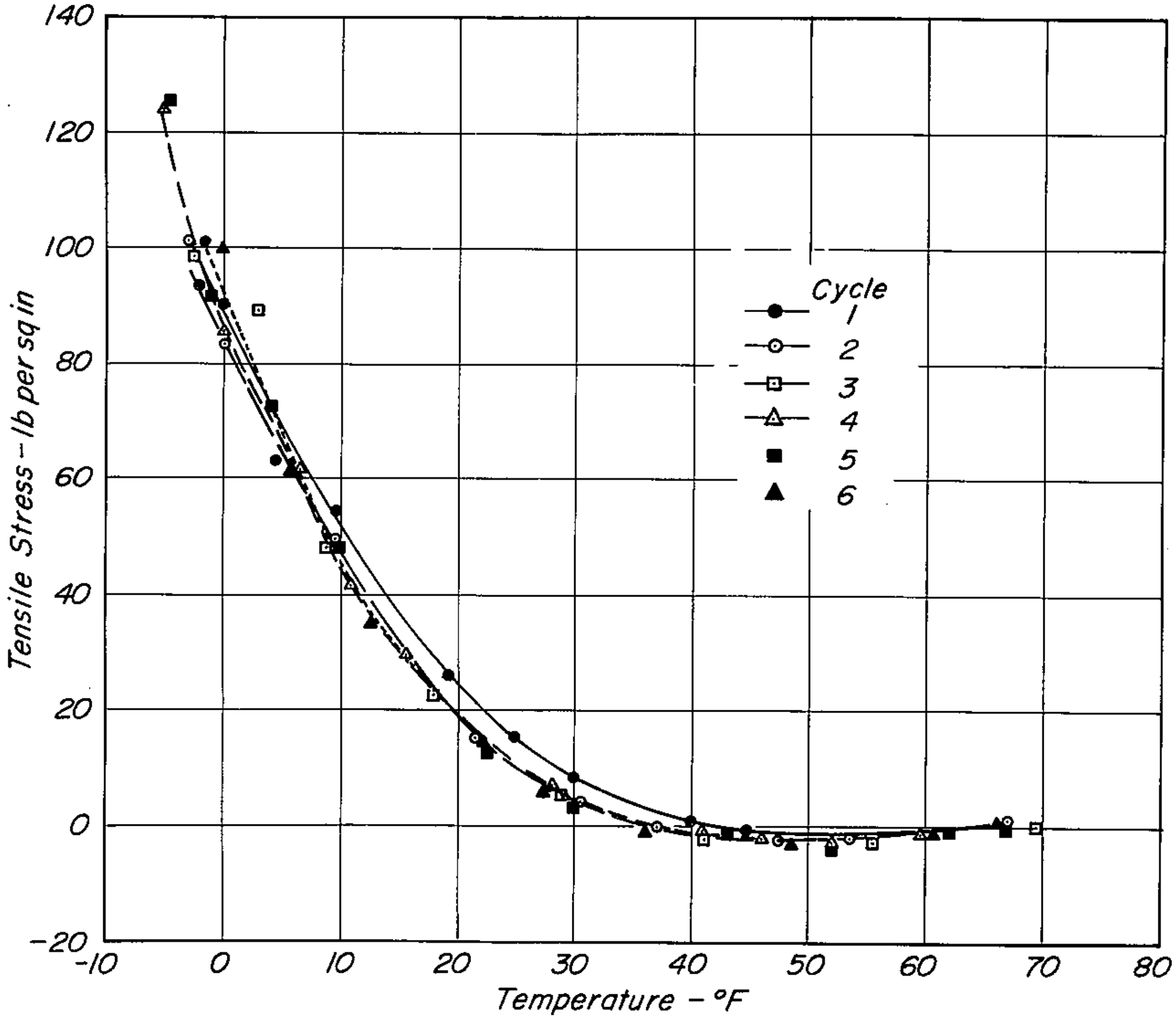


Fig. 20 — Development of tensile stress in laboratory-prepared specimen of asphalt concrete — 6 cycles of cooling, 18°F per hour.

mation that may develop in the asphalt concrete due to temperature change. To further minimize deformations in the frame, it is surrounded by a constant temperature cabinet.

In order to subject the asphalt concrete specimens to temperature change, a small cabinet illustrated schematically in Fig. 17 is placed inside the larger chamber noted above and encompasses 10 in. of the specimen as indicated in Fig. 16. This small cabinet is connected to an external heat source which is capable of providing the required temperature variations around the test specimen.

Briefly, the test procedure is as follows. A particular specimen is placed in the invar frame with its ends inside the end clamps (Fig. 16). The temperature-controlled cabinet is placed over the specimen and the constant temperature cabinet over the entire test frame. The temperature in the inner cabinet is then raised to the starting test emperature of about 140°F to allow the specimen to expand freely before being clamped in position to the frame. At the same time the temperature in the outer (larger) cabinet is brought to a constant value of 100°F to allow the test frame to reach equilibrium. After 4 hours at these temperatures, the outer cabinet is momentarily removed to allow the specimen ends, which project beyond the inner cabinet, to be fixed. This is accomplished by coating the ends of the specimen with an epoxy-resin potting compound. The resin is allowed to cure overnight before the test is initiated with the temperature of both the inner and outer cabinets held constant during this time. An actual test is then conducted by lowering the temperature in the small inner cabinet at any desired constant rate, while recording load (SR-4 load cell—Fig. 16) and temperature. Automatic cycling in the temperature range 140°F to 20°F is possible for any number of cycles.

Results of one cycle of temperature reduction are shown in Fig. 18 for a specimen containing asphalt E at a cooling rate of 11°F per hour. It will be noted that no stress is developed in the specimen until a temperature of less than about 40°F is obtained. Fig. 19 illustrates the results of two cycles of temperature change (cooling—heating—cooling) of 14°F per hour for a specimen prepared with asphalt G. Essentially the same behavior is obtained for asphalt G as with asphalt E. This data would indicate that in the range 140°F to 40°F the asphalt concrete specimens can relax sufficiently fast so that no stress is developed.

Fig. 20 illustrates the development of tensile stress in a specimen of asphalt concrete subjected to six cycles of cooling at 18°F per hour. (The asphalt in this specimen is similar to S-1.) This data illustrates the same general trend as obtained for the project asphalts. It is included to show that it is necessary to obtain temperatures well below 40°F in order to develop appreciable tensile stress—at least for the practical rates of cooling considered herein.

This data is introduced at this time primarily to illustrate that the asphalt concrete, in a practical temperature range and under rates of cooling comparable or even faster than those which might develop in the field, probably exhibits relaxation characteristics sufficient to preclude the development of tensile stresses which might result in thermal cracking.

ACKNOWLEDGMENTS

Personnel associated with the project during this first year include Messrs. Paul M. Wang, Tsuneo Sekine and Glenn A. Secor. Messrs. Wang and Sekine have been responsible for the fatigue tests and Mr. Secor for the development of the thermal stress equipment and test data obtained.

Mr. George Dierking of the ITTE staff prepared the figures.

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